Spring 5-1-1905

The Plate Girder Bridge of the Colorado and Southern Railroad at Boulder, Colorado

Archibald C. Barrett
University of Colorado Boulder

THE PLATE GIRDER BRIDGE OF THE COLORADO AND SOUTHERN RAILROAD AT BOULDER, COLORADO

GRADUATION THESIS

OF
ARCHIBALD C. BARRETT
CANDIDATE FOR DEGREE OF B.S.(C.E.)
UNIVERSITY OF COLORADO.

1905

PREFACE.

****

The work of writing up the Boulder Plate Girder Bridge was suggested by Mr.Howard C.Ford,assistant in Civil Engineering at The University of Colorsdo. It was afterwards suggested to investigste the strength of the bridge and make a thesis of the whole.Whatever success this thesis may win should therefore be credited to Mr.Ford who originated the idea.

The work of watching the progress of the erection was very interesting;it being a great satisfaction to the author to know just the steps followed in replacing a railroad bridge by a more modern structure without interfering with traffic.Thanks is due Mr.W.E.Hague,foreman in charge of the work,for his kindness and willingness to answer questions or to aid in looking up points which he might not know. The investigation of strength was aided very decidedly by the complete set of detail drawings of the bridge which The American Bridge Company kindly furnished.Tracings of these sheets have been placed at the end of the work. The interest of the thesis has been greatly increased by the figures on the costs of the various parts of the work.

These figures were given by Mr. O.J.Travis, Superintendent of Bridges and Buildings of The Colorado and Southern Railroad at Denver, Colorado.

No expression of thanks is adequate for the assistance given by Professor M.S.Ketchum. His willingness to clear up matters not understood was only exceeded by the skill with which he satisfied his questioner.

Archibald C.Barrett.

June 15,1905.

THE COLORADO AND SOUTHERN PLATE GIRDER BRIDGE

ooooooooooo ACROSS ooooooooooo

BOULDER CREEK, BOULDER, COLORADO.

Introduction:- The object of this thesis is to give a detailed description of the method of replacing the old wooden bridge across Boulder Creek by a steel plate girder without interfering with the traffic;also to investigate the strength of the girder in all its parts.

The first part of the thesis includes a description of the bridge, and its abutments; the erection of the superstructure,the manner of riveting, and a summary of the costs.

The second part is an investigation of the strength of the various parts of the steel work according to Cooper's Specifications for Railroad Bridges(1904 Edition.)

---- PART I.-----

Description of Bridge:- The Colorado and Southern plate girder span over Boulder Creek at Boulder was erected in December 1904 to take the place of the two wooden,queen post truss spans which formerly carried the track across the creek.

The bridge now in place was designed and fabricated by the American Bridge Co. at Chicago. It is a through plate-girder,single-track,80 ft. span; 16 ft.,4 ins. deep and made of soft Open Hearth steel. The bridge was designed for Coopers Class E-50 loading. The total weight of the bridge and track is 186,000 lbs. or 93 tons.

Abutments:- The girders are supported on two concrete abutments with wing walls on the up-stream side. They reach 5 ft. above the creek level and extend 15 feet below. The north abutment was placed 10 ft. outside and the south one 20 ft. inside the ends of the old bridge.

Work on the concrete abutments began about the middle of October under the Colorado and Southern foreman in charge of eight men. Sheet piling made of two-inch tongue and groove plank was driven, inclosing the area of the abutments; and as the material was excavated the piling was driven on down. The water was kept out by two horizontal displacement pumps, for which steam was applied by a portable vertical boiler. The material excavated was gravel and boulders, and after a depth of 15 feet was reached without striking bed-rock, the concrete was started. The concrete was put in through a chute and spread and tamped by two men. The composition was:-one part Iola Portland cement, three parts screened creek sand, and five parts broken sandstone, all sizes below 2 1/2 inches. The cement, sand and rock were measured in wheelbarrows, the water was measured in barrels and added during the mixing. The concrete was mixed by turning twice with a shovel.

When the coffer-dams were filled up to the ground level, forms of 2-inch planks were built and the concrete continued up to fill the forms. The top was leveled up by a layer of cement mortar one inch thick.

The concrete was allowed about a month to set before the forms were removed. The total cost was $996.01 for material and $1233.44 for labor.

Erection of Superstructure:- The fabricated steel for the plate girder arrived in Boulder about the time work was begun on the abutments. The girders were packed side by side on two flat cars and were firmly braced and bolted together. The remainder of the steel work was loaded promiscuously.

The erection of the bridge began November 28th.1904. The erection gang consisted of the Colorado and Southern foreman, W.E.Hague and ten laborers. The workmen were inexperienced in bridge work and received $2.50 per day.

The first work was to arrange the old bridge so that the girders could be put in place without disturbing the track and interfering with traffic. In the first place the floorbeams of the old bridge, which were bolted to the under side of the lower chords, were supported on bents made of 12x12 inch posts braced with 2x18 in. planks as shown in the accompanying photograph, Plate I.

When the floorbeams were thus supported, the old wooden trusses no longer carried any load and were unbolted from the floorbeams and were shoved outward about a foot.

PLATE I

Showing method of supporting floorbeams of old bridge.

At the same time the ends of the ties and their supporting stringers were sawed of and removed ,leaving only two feet of tie on each side of the rails, thus making a space so that the girders could be set on the floorbeams .The photograph in Plate II shows men sawing of the ends of the ties on the left hand side,and shows the space left for the girder on the right after the ends have been sawed off.

The girders were then unpacked and set in notches on I2"XI2"sills,6'-0" long, at each end, and braced by two 4"X4" batter posts from the top flange to the ends of the sills .A rope was also thrown over the girderat each end and fastened to the ends of the sills.(See Fig.I)

Plate II.

Showing men sawing off ends of ties.

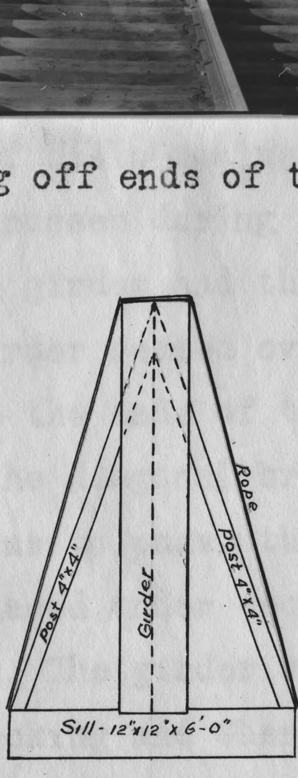


Fig. I

End view of girder packed on trucks.

Each sill was set on a pair of car trucks and separated from them by 3/4" rods laid transversly so the girders would slide easily from the trucks.The first girder was towed into place by a switch-engine and jacked from the trucks on to the blocking supporting each end. A jack was used at each end jacking against the old trusses and the truck-wheels. The blocking was made of ties built up on the floorbeams.

While the girder was being shoved on the blocking two sets of falls hung from the middle of the upper flange were fastened, one on each side, to trees in the creek bottom, and were slackened on one side and tightened on the other as the girder was moved over. The outside set of falls was used to hold the girder against the old trusses during the night. The clearance between the girder and the track was so small that while the girder rested over night on the blocking, the plates on the ends of the bottom flange for connecting with the diagonal bracing had to be cut off to allow trains to pass. the next morning a screw jack was placed under each end of the girders on the floorbeams. The girder was jacked up slightly to loosen the blocking and then lowered as the blocking was removed. The bracing which held the girder on the trucks was fi rst removed. When the girder rested on the floorbeams it was allowed to remain temporarily and the girder on the other side put in in the same way.

It took the erection gang three days to set both girders on the floorbeams, two days being required to place the first girder.

After placing the girders, the remainder of the steel was set on the sides of the track inthe order in which it would be needed; the Colorado &Southern pile-driving outfit with a switch-engine being used to handle the steel.

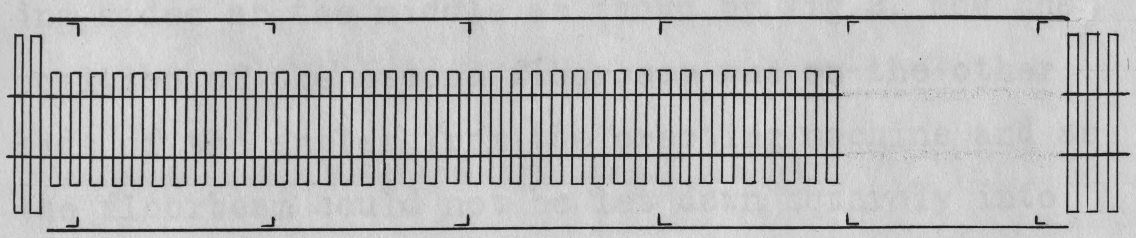
The girders were next supported from the abutments on jacks, the ends of the floorbeams on which they had rested were sawed off and the girders slowly lowered on the abutments. The photograph in Plate III shows men at work sawing off the ends of the floorbeams preparatory to lowering the girders onto the abutments.

PLATE III

Showing men sawing off the floor-beam ends preparatory to lowering the girders to the abutments.

The gang was now ready to put in the floor-system; which consisted of :- four intermediate and two end floorbeams, ten main-track stringers, ten safety stringers, with diagonal and sway bracing. The plan of the floor system is shown in Plate IV.

The floor system was erected a panel at a time beginning at the North end. The old wooden stringers were cut about a foot beyond the second panel point and all the track, ties, and supporting timbers between the cut and the north end were removed, thus leaving an open space to the ground. This work is shown in Fig. 2.



North End.

Fig .2

Showing old track removed from one panel.

The pile diiver then brought up the end floorbeam on the derrick and and let it down into the vacant panel, short lengths of rail being laid so that the track came to the edge of the opening. The end floorbeam was swung around and put in place, the free connecting angles, as will be explained later, were put on and field bolts run through the rivet holes. The second floorbeam was brought up and put in in the same way. However there was considerable difficulty encountered with this one for the following reason :- Each end floorbeam is connected to the girder web by two angles, one on each side of the web of the floorbeam (Plate IV). One of these connecting angles is shop-riveted to the girder and the other is shipped loose, to be field-riveted on after the floorbeam is in place. The shop-riveted angles are on the side of the floorbemm tward the ends of the bridge, changing sides at the middle as shown by Fig.2. Now the position of the second floorbeam was on the other side of the angles from the erecting machine and as the floorbeam could not be let down squarely into position on account of the top flanges of the girders and could not be entered diagonally and turned behind the angles on account of the small space between the angles and the track ahead; one end had to be put in place and the girders spread apart with jacks until the other end could clear its angle and pass behind. When the floorbeam was in place and before it was bolted, it was shoved back slightly and the stringers put in and the whole bolted up. The main stringers could not be put in after the floorbeams were bolted as in swinging around into place they were blocked by the shelf angles of the safety stringers. As soon as the stringers were bolted up, the ties were laid over them, the track put down and trains allowed to pass. In this way all the panels were put in; making two panels a day after the first panel. Work was started after the morning trains had passed, the tracks were relaid for the noon trains; and another panel was put in during the afternoon in time for the evening trains. After three floorbeams were put in making two panels and the fourth floorbeam was reached it seemed as though the errection could go on much faster since the connecting angles were turned the other way and the floorbeams could be set on the same side of the angles as the derrick, but it was soon evident that if the floorbeams were put in their proper places in front of the angles they could not be shoved back as before to allow the main stringers to be put in, so one end of the floorbeam had to be left on the wrong side of the angles until the stringers were in place then jacked by the angles into place as had been done before with the others. Each time when the girders were spread to allow the floorbeam to pass its angle, all the floorbeams in place, save the end one, had to be unbolted at one end to let the girder out.

It seems as if all this labor might have been obviated had the connecting angles been both shop-riveted to the floorbeams, or if the angle which was riveted to one girder had been for the opposite side of the floorbeam to the one on the other girder. The number of field rivets in each case would have been the same. A great saving would also have been made if the rivets in the upper flanges of the girders on the inside and adjacent to the connecting angles had been countersunk; as in every case the floorbeam web plates had to be pried past these heads.

The bridge was erected and bolted together ready for riveting with the exception of the cross frames and diagonals, in ten days after the first girder was brought over or twelve days from the beginning of the work.

Riveting:- On Saturday December 10th. riveting was started. The riveting gang consisted of five men; four riveters and one rivet-heater. The men were entirely new to the work and consequently did very slow work and wasted many rivets. However each day showed an advance in the number of rivets driven and in the quality of the work done. Likewise the men soon learned the tricks of the trade and could make very good heads on very poorly driven rivets.

The first rivets driven were in the east end of the north end-floorbeam as can be easily seen by comparing them with the others. Two men were put in each side of the joint and every hole was drifted before receiving the rivets. The field bolts which were in every other hole were removed as the riveting progressed.

The method of riveting was as follows:- The rivet heater ran over with a hot rivet in his tongs, man No.2 took it in his tongs and set it in the hole, No.3 drove it up, No.2 set a dolly,or heavy iron maul with a die in the head, over the rivet head and Nos. 2 and 3 pulled back on the dolly handle against a hook, bolted in the hole above, which held the handle near the head and acted as a fulcrum. This held the rivet head tight while No.4 and No.5 headed it upon the other side. No.4 first upset the rivet thus spreading it and filling the hole. No.5 then placed his "snap"over the end and No.4 hammered the "snap" till the head was formed.

The first day the end of the floorbeam was riveted up, 44 rivets being driven and 8 rivets wasted.The second day 118 rivets were driven and six wasted.

While the riveters were at work the other men were bolting up the cross frames and diagonal bracing,clearing away the old timbers and putting in the cross ties. All but the riveters stopped work on December 18th.,the latter working on till January 10th. In all 1549 rivets were driven. The time occupied in driving the rivets was 27 days or an average of 57 1/2 rivets per day. With five men at $2.50 per day or $12.50 per day for the gang, the rivets cost $\left(\frac{12.50 \times 27}{1549} = 22\right)$ 22 cents a piece, which is a very high cost, 10 to 15 cents being an average cost.

A criticism on the rivet driving is that the lengths of the rivets were not determined from the drawings but were obtained by trial. For instance in riveting the connecting angles of the intermediate floorbeams to the girders, 3 inch rivets were used while the plans required 3 5/8 inch rivets. This showed that the rivets were not "upset" sufficiently to fill the holes before heading them up. This was apparent from the method of heading up. When a hot rivet came through it was first bent over against one side of the hole, then was hit a few raps on the head supposedly to fill the hole, but in reality only mashing the end against the outside of the hole, after which the snap was put on and the head formed.

The cost of erecting the bridge was as follows:-

Material-----	$129.70
Labor ------	869.55
Total ------	$999.25

The total weight of the bridge without track was 155195 lbs. or 77.6 tons. Then the cost of erection per ton was $\frac{999.25}{77.6} = 12.87$ dollars. An average cost for such work is from $6 to $10 per ton.

Summary of Costs

	Material	Labor
Excavating and putting in two abutments,	$996.01	$1233.40
Erecting and riveting steel girders,	129.70	869.55
Cost of girder with inspection at bridge works and freight from Chicago,	4178.93	54.95
Labor cleaning up and filling approaches,		1450.00
Totals,	$5304.64	$3607.90
	3607.90	
Total cost,	$8912.54	

PART II.
****

pecifications:- This investigation is to be carried on according to Cooper's Specifications for Railroad Bridges,I90I Edition, and the following loadings will be used as they are the ones for which the bridge was designed.

Live Load: Cooper's Class E-50.

Dead Load:-Weight of metal,2000 lbs.per ft.

" " track,400 " " ".

Wind Load:-Fixed wind load,300 " " ".

Moving " " ,450 " " ",

epth and Spacing:- The depth of girders varies in common practice from I/I0 of the span to I/I2,an average ratio being I/I0.5 In this bridge the ratio is 8 I/3 :80 orI/9.6 The distance C to C of girders is I6'-4". Cooper specifies a clearance of I4ft.and an additional clearance in inches of .8 the degree of the curve,on each side,for curvature. The track over this bridge has a(8°24)degree curvature so requires an additional clearance of .8X8.4 = 6.7 or a total specified clearance of I4+1.3 = 15'-3"

------ Floor System.------

Estimate of Weights

The weight of an intermediate floorbeam, as marked on the piece, is 4100 lbs. An intermediate floorbeam differs from an end floorbeam in the following parts:-

Piece	No. of pieces	Cross Section	Length in ft.	Weight per ft.	Total Weight
L s	4	6X6X3/4"	15.302	5.1	312.
Gus.pl.	2	$15\frac{1}{2}$"X$\frac{1}{2}$"X$7\frac{1}{2}$"	.656	26.35	33
L s	4	3"X3"X3/8	7.2	.656	19
spl.pls	4	12 1/2"X 1/2	.333	21.25	28
Fills.	4	14"X 3/4"	.167	35.7	24
Fills.	4	12 1/2"X3/4"	.167	31.88	21
L s	2	$3\frac{1}{2}$"X 5"X $\frac{3}{8}$"	.1	10.4	21
L s	2	$3\frac{1}{2}$"X 5"X $\frac{3}{8}$"	1.5	10.4	31

Difference in weight = 489 lbs

Weight of intermediate floorbeam = 4100 "

" " end " = 3611 "

Safety Stringers.

Piece	No. of pieces	Cross Section	Length in ft.	Weight per ft.	Total Weight
I beam	1	20"	15.292	65.	994
Con L s	4	6"X6"X1/2"	1.458	19.6	114
Fill pl	1	6 1/4X1/2"	0.333	10.63	4
Rvt.Hds	28	7/8"		$\frac{24.29}{100}$ ea	7

Total weight = 1119 lbs

Piece	No of pieces	Cross Section	Length in ft.	Weight per ft.	Total weight
Main stringers					
Web pl.	1	31 3/4"X1/2"	15.29	53.98	825
Fl. ∟s	4	$3\frac{1}{2}$X5"X 5/8"	15.32	16.8	1030
Con. ∟s	4	6"X6"X 1/2"	2.56	19.6	206
Fill pls	4	9"X 5/8"	2.08	19.13	159
Stif. ∟s	4	$3\frac{1}{2}$X$3\frac{1}{2}$X$\frac{3}{8}$"	2.56	8.5	87
Fill pl	4	3 1/2"X 5/8"	2.0	7.44	59
Con.pl.	1	12"X 3/8"	2.00	15.3	31
" "	1	9"X 3/8"	1.5	11.48	17
Rvt.Hds	358	7/8"		$\frac{24.29}{100}$	87

Total weight = 2501 lbs

Stringer Brackets.

Piece	No of pieces	Cross Section	Length in ft.	Weight per ft.	Total weight
Fl. ∟s	2	$3\frac{1}{2}$"X5X $\frac{5}{8}$"	1.166	16.8	39
Web pl.	1	13 3/4"X1/2"	2.646	23.38	62
Fill pl	2	4"X 5/8"	2.354	8.5	40
Con. ∟s	2	4"X6"X1/2"	2.615	16.2	85
Fill pl	1	4"X1/2"	0.875	6.8	6
Rvt.Hds	26	7/8"		6.8	6

Total weight = 238 lbs

Safety Stringer Brackets.

Piece	No of pieces	Cross Section	Length in ft.	Weight per ft.	Total weight
I beam	1	20"	1.166	65	76
Con ∟s	2	6"X6"X 1/2"	1.458	19.6	57
Fill	1	6 1/4"X 1/2"	0.333	10.63	4
Rvt.Hds	14	7/8"		$\frac{24.29}{100}$	3

Piece	No of Pieces	Cross Section	Length in ft.	Weight per ft.	Total Weight
Total weight of safety stringer brackets					140 lbs

****

:Cross Frames.

Ls	2	3"X3"X3/8"	4.771	7.2	68.6
Ls	2	3"X3"X3/8"	4.193	7.2	60.4
Pls.	4	9 1/2"X3/8"	1.167	12.12	56.6
Rvt.Hds	50	7/8"		24.29/100	12.1
		Total weight		=	197.7

****

:Stringer Laterals.

L	1	3"X3"X3/8"	5.599	7.2	40.
		Total weight		=	40. lbs

****

:Bottom Laterals, BL_1 & BL_2.

L	2	$5X3\frac{1}{2}"X\frac{1}{2}"$	1.035	11.1	388.8
L	8	$5X3\frac{1}{2}"X\frac{1}{2}"$	20.135	13.6	547.4
L	8	$3\frac{1}{2}"X3\frac{1}{2}"X\frac{3}{8}$	1.125	8.5	76.5
Pl.	8	9"X3/8"	1.125	11.48	103.3
L	2	$5"X3\frac{1}{2}"X\frac{1}{2}"$	9.776	13.6	266
L	2	$5"X3\frac{1}{2}"X\frac{1}{2}"$	9.88	13.6	268.8
L	4	$3\frac{1}{2}"X3\frac{1}{2}"X\frac{1}{2}"$	1.125	11.1	50.
Rvt.Hds	240	7/8"			58.3
		Total weight		=	1459.1

****

Bottom Laterals, BL3&BL4.

Piece	No. of Pieces	Cross Section	Length in ft.	Weight per ft.	Total Weight
L	2	$3\frac{1}{2}$"X$3\frac{1}{2}$"X $\frac{1}{2}$"	20.219	11.1	449.8
L	8	" X " X $\frac{3}{8}$"	1.125	8.5	76.5
L	2	" X " X $\frac{1}{2}$"	9.943	11.1	22o.6
L	2	" X " X "	9.901	11.1	220.0
Plate	8	9"X 3/8"	1.125	11.48	103.3
L	4	$3\frac{1}{2}$"X$3\frac{1}{2}$"X $\frac{1}{2}$"	0.625	11.1	27.8
Plate	2	7 1/2"X 3/8"	1.875	9.57	57.9
Rvt.Hds	152	7/8"			36.9
		Total weight			= 1170.8

****

Bottom Laterals, BL_5&BL_6.

Piece	No. of Pieces	Cross Section	Length in ft.	Weight per ft.	Total Weight
L	1	$3\frac{1}{2}$"X$3\frac{1}{2}$"X $\frac{3}{8}$"	20.219	8.5	172.0
L	1	" " "	9.943	8.5	84.4
L	1	" " "	9.901	8.5	84.1
L	4	" " "	1.125	8.5	38.2
L	2	" " "	0.625	8.5	10.6
Pl.	4	9"X 3/8"	1.125	11.48	51.6
Pl.	1	7 1/2"X 3/8"	1.625	9.57	15.5
Rvt.Hds	72	7/8"			17.5
		Total weight			= 473.9

****

Table. 1

Summary of Weights of Floor System.

No.of pieces	Name of piece	Weight each	Total weight
2	End floorbeam	36II	7222
4	Int.floorbeam	4I00	I6400
I0	Main Stringer	250I	250I0
10	Safty Str.	1119	11190
4	Str.bracket	238	952
4	S.Str.bracket	140	560
10	Cross frame	197.7	1977
15	Str.lateral	40	600
2	Bot.laterals: B.L.1--B.L.2	1459.1	2918
2	B.L.3--B.L.4	1170.8	2342
1	B.L.5	473.9	474

Total weight of floor-system : 69645 lbs. :

****

Stringers.

The main stringers are spaced 5'-0" C to C and the centers of the safty stringers are 2'-0" outside of the main stringers.

The main stringers are designed to carry all the load and the safty stringers are made half as strong as the main stringers.

Dead Load:-Weight of main stringer = 2500 lbs.

Cross-frames & bracing= 500 " .

Track @ 400 lbs.per ft. 3100 " .

Total stringer dead load = 6100 " .

Dead load moment = $\frac{6100 X 15.5 X 12}{8}$ = 141825 in.lbs.

Live load moment (Cooper pg. 30) =1987500 " "

Total moment = 2129325 " "

conomic Depth :- Johnson's formula for economic depth is ,$X = 1.41\sqrt{\frac{m}{f\ t}}$

m = total moment = 2129325 in.lbs.

f = allowable stress on gross area = 85%of 9000 lbs or approximately 7650 lbs.

t = thickness of plate = 1/2 inch

Then $X = 1.41\sqrt{\frac{2129325}{7650 X .5}}$ = 33.3inches

The actual depth of the stringer web plate is 31.75 inches which is not economic according to this formula. Practice however uses a less depth than this formula.

The effective depth is 32 - 2 X.95 = 30.1 inches

NOTE:--The allowable stress for soft steel in tension is 9000 lbs.per sq.in

hear in the web:- Live load end ahear = 51560 lbs.

Dead load end shear = 3050 "

Total end shear = 54610 "

The gross section of the web is 31.75 X.5 = 15.87 sq.in.

Sectional area of 9 rivet holes = 4.5 "

Net area of web plate-------- = 11.37 "

Unit end shear in web plate = $\frac{54610}{11.37}$ = 4800 lbs./sq.ft.

Allowable unit shear (Lewis'Specifications) for soft steel is 5000 lbs. Therefore the section is ample.

anges:- The flange stress is $\frac{2129325}{30.1}$ = 70800 lbs.

Area required = $\frac{70800}{9000}$ = 7.67 sq.in.

The flanges are made of 2-angles 3-1/2"X 5"X 5/8"

the area of which = 9.86 sq.in.

iffeners:- The specifications call for stiffeners if the shearing stress exceeds $10000-75\frac{d}{t}$ =in this case,6190 Since the end shearing stress is 4800 lbs.,stiffeners are not required. There is however a pair of stiffeners five ft. from the end and connected by 8 rivets.

onnecting Angles:- The connecting angles are:

2-angles 6"X6"X 1/2" and are therefore of ample section to resist bearing.

ivet Spacing in Flanges:-

Rivet spacing =P = $\frac{rh}{s}$ (Mill Buildings pg.183)

r = allowable bearing on one rivet.

h = distance between rivet lines.

s = maximum shear.The maximum end shear = 54610 lbs.

This value is taken as produced by a uniform load, erring on the safe side,and the shear at different sections computed.

Req'd spacing at end = $\frac{4680 \text{ X } 28}{26400}$ =2.4 inches.

Actual spacing = 2.63 inches up to a point from the end 4'-1-7/16".

Req'd spacing at 4' point = $\frac{4680 \text{ X } 28}{26400}$ = 4.96 inches.

The maximum spacing in the stringer is 5" at the middle so the rest of the spacing is within the limit.

Rivet Spacing in Connecting Angles between Angles and Web Plate:-

The end shear per stringer =54610 lbs

The number of rivets required in the stringer in bearing = $\frac{54610}{4680}$ = 11.7 and their are 13 rivets spaced in two rows.

Floorbeams

The floorbeams are spaced 15-5"C to C.

Live load:- The live load maximum floor-beam reaction is 69687 lbs.per stringer(Cooper pg.30)

The live load moment = 69687 X 12 X 5-2/3 = 4738716in.lbs

(The centers of main stringers are 5'-8"from the centers of the girders.)

Dead load:- Weight of floorbeam	=	4100 lbs.
" " stringer	=	2501 "
" "Crossframes etc.	=	500 "
" " Track	=	3100 "
" " Safty Stringer	=	1119 "

Moment of main stringer and track =

(2501+500+3100)X12X5-2/3 = 414800 in.lbs.

Moment of safty Str.=1119 X 12 X 3-2/3=47000 " "

" "Floorbeam $=\frac{4100X12X16-1/3}{8}$= 100450 " "

Total dead load moment = 562250 " "

" live " " = 4738716 " "

" combined " = 5300966 " "

conomic Depth:- $X = 1.41\sqrt{\frac{5300966}{7650X1/2}} = 52.2$ inches.

The actual depth is 42.25 inches.

The effective depth is 3'-6-1/2"minus 3-1/2"= 39"

langes:- The flange stress is $\frac{5300966}{39} = 136000$ lbs.

The rivets in the flanges are spaced alternately in two rows (Fig.3) and are so close that the diagonal distance betweenadjacent rivets does not exceed by 30% the transverse distance between rivet lines, therefore the rupture must be considered along the diagonal line and the area of two rivet holes must be deducted from the gross area of each angle.

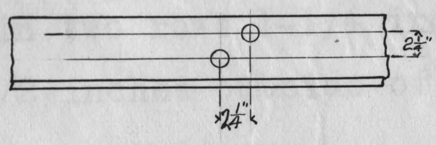


Fig. 3

Showing section of spacing in floorbeam flange.

The flange angles are 2 angles 6"X6"X3/4"

whose area together is	16.88	sq.in.
deduct 4 rivet holes or	3.00	" "
leaves a net section of	13.88	sq.in.

The unit stress is $\frac{136000}{13.88} = 9730$ lbs.per sq.in.

The allowable stress is 9000 lbs. per sq. in.

ivets for Connecting Angles,between Stringer &Floorbeam:

The live load floorbeam reaction	= 69637	lbs.
" Dead " shear	= 6100	"
Total end "	= 75787	"

The number of rivets required is $\frac{75787}{3150} = 24.1$

The actual number is 22.

The same criterion determines the number of rivets in the connecting angles between the angles and the web of the floorbeam.

ivets between Connecting Angles and Web of Girder:-

One angle is shop riveted and the rivets are in single shear, so the allowable stress will be taken as 3600 lbs. The number required is $\frac{77837}{3600} = 22$

The actual number is 23.

ivet Spacing in Flanges:-

$$p = \frac{rh}{s} = \frac{4680X36}{77837} = 2.17 \text{ inches at the ends.}$$

The rivets are in two rows 1-1/4 inches apart and the pitch is 2-1/2 inches outside of the stringers and 4 inches between stringers.

hear in Web:- The gross section of the web is

42.5"X1/2"	=	21.25 sq. in.
deduct 10 rivets	=	5.0 " "
Net area	=	16.25 " "
Maximum end shear	=	77837 lbs.

Unit stress in web = $\frac{77837}{16.25}$ = 4780 lbs.per sq.in.

The allowable stress is 5400 lbs.per sq.in.

The section is therefore ample.

End Floorbeam

The web section is the same as for the intermediate floorbeam so it is more than sufficient.

Live Load:- The live load shear is 103124 lbs. (Cooper pg.30)

The live load moment = $\frac{103124}{2}$ X 12 X 5-2/3
= 3507000 inch lbs.

Dead Load:- Moment of stringer and track =
3050X12X5-2/3 = 207400 in.lbs.

Moment of safty str.= $\frac{1119}{2}$X12X3-1/3	= 23500	" "
" "Floorbeam = $\frac{3611X12X16-1/3}{8}$	= 88400	" "
Total dead load moment	= 319300	" "
" live " "	=3507000	" "
" combined "	=3826300	" "

The effective depth is 42.5 - 2.16 = 40.3 inches.

The flange stress is $\frac{3826300}{40.3}$ = 94940 lbs.

The flange angles are 2 angles 6"X 4"X 3/4"

whose area =	13.88	sq.in
Deduct 2 rivet holes	1.5	" "
Net area =	12.38	" "

The unit stress is $\frac{95200}{12.38}$ = 7680 lbs per sq.in.

The allowable unit stress is 9000 lbs. per sq.in.

Therefore the flanges are of sufficient strength.

Girders.

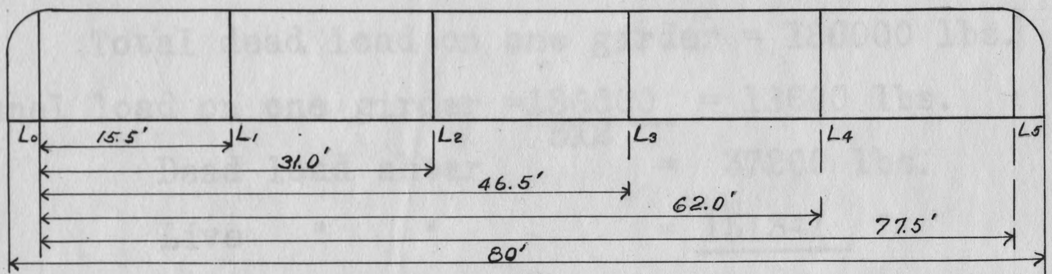


Fig. 4.

Web Section:-

Live Load:- Maximum live load end shear in panel L_0L_1 = 151341 lbs.(Cooper p-30)

Dead Load:- Assumed weight of metal = 2000 lbs.per ft.

" " " track = 400 " " "

(per ft.of bridge") Dead Load = 2400 " " "

or 1200 lbs.per linear ft.of girder.

Total dead load = 2400X77.5 = 186000lbs.

minus weight of track = 400X77.5 = 31000 "

leaves the assumed weight of steel = 155000 "

This does not include the ends of the girders which extend beyond the supports.

The actual weight of the bridge steel is:-

Weight of two girders = 85550 lbs.

Total weight of floor-system 69645 "

Total weight of bridge steel = 155195 "

This value differs from the estimated weight by only 195 lbs. which certainly is a remarkably close estimate. The assumed weight or 155000 lbs. will be used in this investigation.

Total dead load on one girder = 186000 lbs.

Panel load on one girder $= \frac{186000}{5X2} = 18600$ lbs.

Dead load shear	=	37200 lbs.
Live " "	=	151341 "
Total end shear	=	188541 "

The unit shearing stress is taken as 7200 lbs. then the required net section of the web is 99.5"X 3/8"= 37.3 sq, in.

Area of 23 rivet holes	=	8.6 " "
Net area	=	28.7 " "

Sectional Area of Flanges:- The maximum live load moment (Cooper pg. 30) is -----------2551406 ft. lbs.

Maximum dead load moment =

$\frac{93000X77.5}{8}$ =-------- 901700 " "

Total moment = 3453106 " "

or 41437272 inch lbs.

Composition of Flanges:- The flanges are made up as follows:-

2 angles 8"X8"X3/4", area	=	22.88 sq. in.
1 cover plate 18"X5/8"	=	11.25 " "
1 " " " "	=	11.25 " "
1 " " 18"X1/2"	=	9.0 " "
Total gross area	=	54.38 " "

rea of Rivet Holes Deducted:-

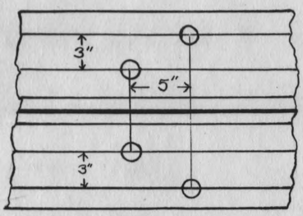


Fig.5.
Smallest rivet spacing in lower flange near center of girder.

Considering rupture along a diagonal line of holes, the path between center of rivets is $\sqrt{5^2+3^2}$ = 5.83 in

- 1.0 "

" " " edges of holes -------- = 4.83 "

The distance between rivet lines is = 3.0 in.

half of a rivet hole = 0.5 "

The transverse line across rivet holes = 2.5 "

Since 4.83 exceeds 2.5 by more than 30%, the transverse line will be considered and two rivet holes will be deducted from each angle and each plate.

Areas of holes deducted:-

4 holes in angles at .75 sq.in. each	= 3.00	sq.in.
" " " 2 plates at .63 " " "	= 2.52	" "
2 " " 1 " " .5 " " "	= 1.0	" "
Total rivet hole area	= 6.52	" "
Gross area	=54.38	" "
Total net area of flanges	= 47.86	" "

fective Depth of Web:-

Center of gravity of angles from backs = 2.19 inches

" " " plates " same = 0.875 "

Call area of angles ----- A_1

" " " plates ----- A_2

Center of gravity of gross flange from angle backs is $\frac{A_1 X\ 2.19--A_2 X 0.875}{A_1 + A_2} = \frac{50 - 27.5}{54.38} = 0.414$ inches.

Twice this value or .828 inches added to the depth of web. plate = 100.0 inches gives an effective depth of 100.83 inches. The unit stress for soft steel in tension is 9000 lbs. therefore the area required is $\frac{M}{SH} = \frac{41437273}{9000X100.83} = 45$ sq. in.

The section is therefore of sufficient area.

Web Splices:- the web of the girder is spliced at each panel point and the splices are all similar. The splice requiring the greatest strength is that at L_1 and it will therefore be investigated. Since the loads are carried to the girders at the panel points by the floorsystem in a through girder, the shear must be considered as constant between panels. Consider the shear at L_1 as that in the panel L_1L_2.

Live Load Shear in Panel L_1L_2:-

Try wheel 2 of Cooper's E-50 loading, at L_2, then the total load on the girder is 142000 lbs. The average panel load is $\frac{142}{5} = 28400$ lbs. The load in the panel L_1L_2 is 10 to 30 thousand pounds.

This position therefore produces a maximum shear. he rule is :- The maximum live load shear occurs when the load in the panel is equal to the total load on the bridge divided by the number of panels. With the loads in this position ,wheel number 9 is 6.5 ft. from the right end of the bridge and the shear in

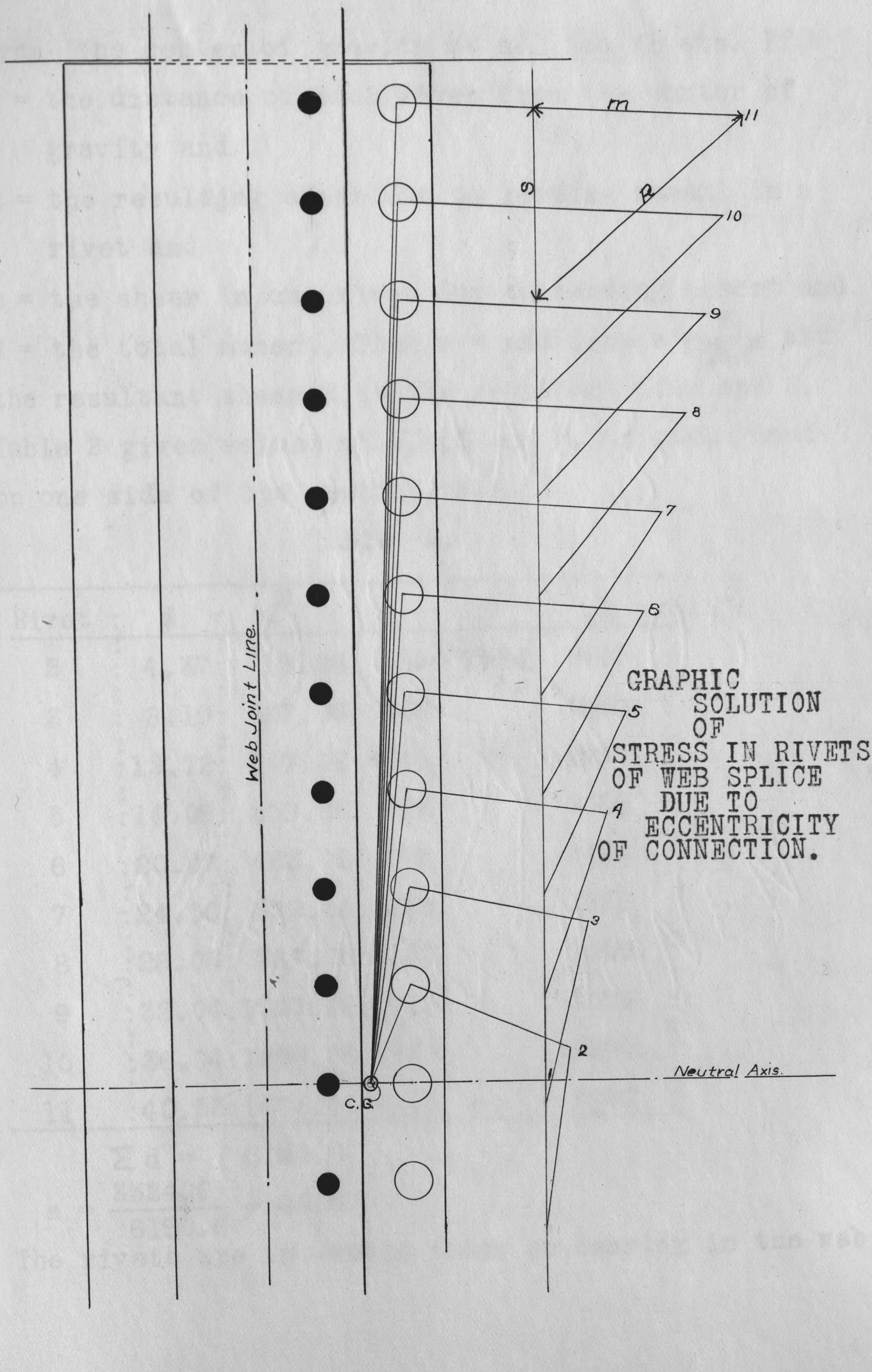
$$L_1\,L_2 = \frac{3496 + 142 \times 6.5}{77.5} - \frac{80}{15.5} = 51600 \text{ lbs. for E-40}$$

or for E-50 loading $\quad = 64500$ lbs.

The dead load shear in $L_1\,L_2$ $\quad = \underline{18600}$ "

Total shear in $L_1\,L_2$ $\quad = 83100$ "

The splice consists of two plates,one on each side of the web,14-1/2X3/8"and consists of two rows of 21 rivets on each side of the joint. The plates are therefore of ample section to transmit the shear,so that the riveting is the only thing needing investigation. One of these rows of rivets is of field rivets holding one connecting angle from the floor-beam. The direct shear taken by each rivet is S $= \frac{83100}{42} = 1978$ lbs. Besides the direct shear,each rivet must take shear due to moment,caused by the eccentricity of the connection. The center of the rivet lines is 4 inches from the line of action of the direct shear,therefore the total moment of the direct shear is $M = 83100 \times 4 = 332400$ inch lbs. Each rivet will take an additional shear due to its share of this moment which varries as its distance


m
11
S
R
10
9
8
7
6
5
4
3
2
1
Web Joint Line.
Neutral Axis.
C.G.
GRAPHIC
SOLUTION
OF
STRESS IN RIVETS
OF WEB SPLICE
DUE TO
ECCENTRICITY
OF CONNECTION.

from the center of gravity of all the rivets. If
d = the distance of each rivet from the center of gravity and
a = the resulting shear due to bending moment in a rivet and
m = the shear in any rivet due to bending moment and
M = the total moment, Then m = aXd ,and $a = \frac{M}{\Sigma d^2}$ and the resultant shear, R, is the resultant of m and S. Table 2 gives values of d, m, S and R for each rivet on one side of the neutral axis.

Table 2.

Rivet	d	d^2	m	S	R
2	4.37	19.06	234	1978	1900
3	8.19	67.06	438		1960
4	12.12	147.06	648		1980
5	16.09	259.06	862		2050
6	20.07	403.06	1078		2180
7	24.06	579.06	1288		2280
8	28.05	787.06	1500		2400
9	32.04	1027.06	1715		2520
10	36.04	1299.06	1930		2700
11	40.03	1603.06	2143		2850

$\Sigma d^2 = 6190.6$

$a = \frac{332400}{6190.6} = 53.6$

The rivets are in double shear so bearing in the web will govern. The bearing values of 7/8" rivets are:- for shop rivets 4410 lbs., for field rivets 2970 lbs. The bearing of each rivet is thus less than the allowable, and since this splice is efficient the others must be alright.

eb Stiffners:- Cooper specifies the use of stiffners where the unit shearing stress exceeds that allowed by the formula:- $S = 1000 - 75 \text{ X } H \qquad H = d/t$

then $S = 1000 - 75 \text{ X } \frac{99.5}{3/8} = 10000 - 19875 = -9875$

The unit shear exceeds this amount so stiffners are required. All stiffners must be capable of carrying the maximum vertical shear without exceeding the allowed unit stress:- $P = 10000 - 45 \text{ X } l/r$

The stiffner angles in the first panel are $3\frac{1}{2}$" X 5" X $\frac{1}{2}$" X 100". The area of the two is 8.0 sq.in. and $r = 2.43$. Then $P = 10000 - 45\frac{100}{2.43} = 8155$ lbs.

The total shear in the first panel is 145700 lbs. then the stiffening area required is $\frac{1457000}{8155} = 17.87$ sq.in. This is more than twice as large as the actual area, but in practice the stiffners seldom come up to Cooper's Specifications. Johnson ptonounces stiffners sufficient if the outside legs are 1/30 of the depth of the web. According to this, these stiffners are ample.

In panel 2 the stiffners are 3/8" thick and the area is 6.1 sq.in. The total shear is 83100 lbs.

making the required area = $\frac{83100}{8155}$ = 10.2 sq.in.

This also is insufficient --------according to Cooper

In panel 3 the stiffeners are 3/8"thick and must take only the live load shear which is 31,750 lbs., giving a required area of $\frac{31750}{8155}$ = 3.89 sq.in.These stiffners are therefore ample.

The stiffners are spaced at distances varying from 5'-0$\frac{1}{2}$" to 5'-3$\frac{1}{4}$ which just exceeds Cooper's rule by the odd inches.

At the ends of the girders at panel points L_0 & L_5 the web is stiffened by :-

4 angles 3 $\frac{1}{2}$" X 5" X $\frac{1}{2}$"	area	=	16.	sq.in.
2 fills 7 $\frac{3}{4}$" X $\frac{3}{4}$" -------	"	=	11.62	" "
1 " 4 $\frac{3}{8}$"X $\frac{1}{2}$"-------	"	=	2.38	" "
Total area		=	30.0	" "

Besides this the upper flange angles are bent down over the ends of the girder and take some shear, so the end shear at L_0 is amply provided for.

Flange Plates:-"Flange plates must extend beyond their theoretical length ,two rows of rivets at each end."
" Where two or more plates are used in the flanges,they shall be either of equal thickness or shall decrease in thickness outward from the angles."

Johnson gives the following formula for the theoretical lengths of cover plates:-(pg.302)

$$X_n = L\sqrt{\frac{a_1 + a_2 + --- a_n}{A}}$$

where X = length of plate; l = the span; $a_1, a_2, -- a_n$, = areas of 1st,2nd,--nth,etc.plates from outside; A = total area. The theoretical and actual lengths of the plates are given in Table 3.

Theoretical length	+4 rows of rivets	Actual length
$X_1 = 77.5\sqrt{\frac{8}{47.88}} = 31.6$	32.6	32.6
$X_2 = 77.5\sqrt{\frac{18}{47.88}} = 47.5$	48.5	51.1
$X_3 = 77.5\sqrt{\frac{28}{47.88}} = 59.4$	60.7	60.7

Table 3.

The plates are therefore of the right lengths.

ivet Spacing in Girder Flanges:-

Shears in the panels.

Dead load shear	37200	18600	0
Live " "	108500	64500	31750
Total "	145700	83100	31750

Table 4.

Rivet spacing = $p = \frac{rh}{s}$. r = 4410, h = 91, s = values given in table 4.

Theoretical and actual spacing in panels 1,2,3.

Theoretical spacing	Actual spacing
$p_1 = \frac{4410X91}{145700} = 2.75$ inches	3 inches
$p_2 = \frac{4410X91}{83100} = 4.825$ "	4 "
$p_3 = \frac{4410X91}{31750} = 12.63$ "	5 "

Table 5.

Thus the actual spaclng is in efficient in the first panel.

Rivet Spacing in Flange Plates:- The proportions of flange stress taken by one, two and three plates respectively are approximately, 26, 41, and 51 per cent.

$p = \frac{rh}{s}$. $r = 4860$ (rivets are in single shear.)

$h = 100, 101.25,$ and $102.5.$

$s = .26X145700, .41X83100,$ and $.5X31750.$

Spacing in first panel for first plate.

$$p_1 = \frac{4860X100}{145700X.26} = 12.8 \text{ inches.}$$

Spacing in first panel for second plate.

$$p_2 = \frac{4860X101.25}{145700X.41} = 8.25 \text{ inches.}$$

Spacing in second panel for third plate.

$$p_3 = \frac{4360X102.5}{83100X.51} = 11.8 \text{ inches.}$$

Thr largest actual spacing in the flange plates is 6 inches and this is less than the minimum which is necessary, so all the spacing is within the limit. The actual spacing however is very irregular due to the necessity of passing of the plate joints, stiffners and floorbeams in an easy, symmetrical manner; also since

Laterals.

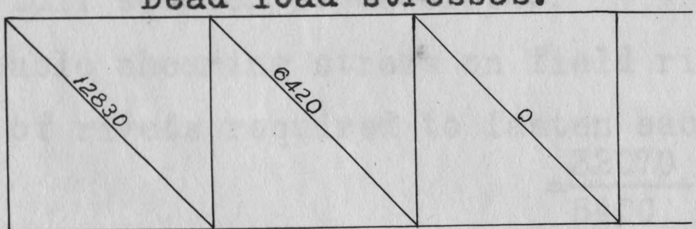
Wind Load:- The fixed wind load is 300 lbs.per foot making the dead panel load = 300X15.5 = 4650 lbs. The live load is 450 lbs.per ft.or 6975 lbs.panel load.

W = dead panel load = 4650 lbs.

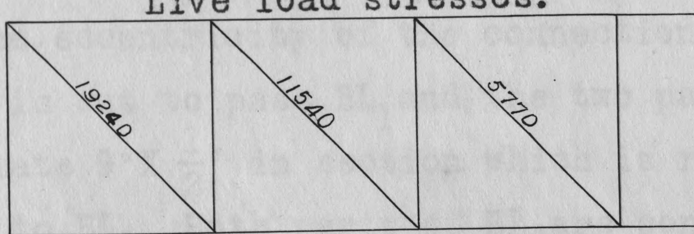
P = live " " = 6975 lbs.

$\theta = 43°30'$, W sec.θ = 6420, P sec.θ = 9625.

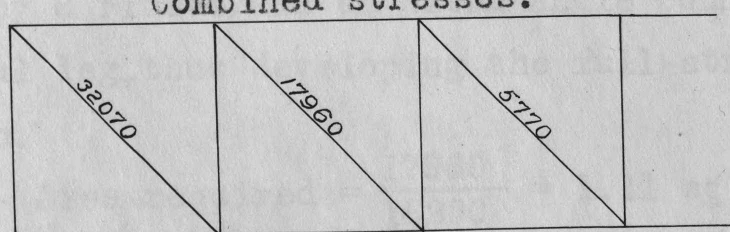
Dead load stresses.



Live load stresses.



Combined stresses.



BL_1 & BL_2:- Allowable tensile stress on field rivets = 16200 lbs.per sq.in.

Area required = $\frac{32070}{16200}$ = 1.98 sq.in.

Gross section = 1 angle 5"X $3\frac{1}{2}$"X $\frac{1}{2}$" = 4 sq.in.

Deduct two rivet holes = 1 " "

Net section = 3 " ".

The angles are fastened by both legs so the strength of the full section is developed.

Allowable shearing stress on field rivets = 5400 lbs

Number of rivets required to fasten each end = $\frac{32070}{5400}$ = 5.95

The actual number is 8 which is sufficient to allow for the eccentricity of the connection.

BL_2 is cut to pass BL_1 and the two parts are joined by a plate 9"X $\frac{1}{2}$" in section which is riveted by two rivets to BL_1. Both parts of BL_2 are connected to the plate by 8 rivets and an extra angle connecting the vertical leg, thus developing the full strength of the section.

BL_3 & BL_4:- Area required = $\frac{17960}{16200}$ = 1.11 sq in.

Gross section = 1 angle 3 1/2"X3 1/2"X1/2"= 3.25 sq.in

Only one leg of this angle is connected so the area considered is only $\frac{1}{2}$ of the above = 1.62 sq.in.

Deduct one rivet = .5 " "

Net area = = 1.12 " "

Number of rivets required = $\frac{17960}{5400}$ = 3.3

There are 5 rivets and they are in dirrect shear.

L_5& BL_6:- Area required = $\frac{5770}{16200}$ = .356 sq.in.

Gross area = 1 angle 3 1/2"X3 1/2"X3/8" = 2.49 sq.in.

Deduct one rivet	= .37 " "
Net area	= 2.12 " "

The angles are connected by only one leg so only one half of the section is counted, or 1.06 sq.in.

Number of rivets required at the ends = $\frac{5770}{5400}$ = 1.+

There are 4 rivets in dirrect shear.

In the case of the lateral, in each panel, which is cut; the ends are joined by a plate of sufficient section, by means of an extra angle to connect the vertical leg, and as many rivets are used as are at the ends of the laterals.

Conclusion:- Each part of the bridge has now been investigated. Most of the parts have stood the tests, but the following have not come up to the specifications:

1. The rivet spacing in the ends of the stringer flanges exceeds by .22 inches the theoretic pitch. The pitch however is the minimum allowed. This shows, as did the investigation of the depth of the web, on page , that the depth of the stringer is insufficient.

2. The depth of the floorbeams is 10 inches less than the economic depth.

3. The unit tensile stress in the floorbeam flanges is slightly greater than the allowable stress.
4. There is an insufficient number of rivets,by3, between the stringer connecting angles and the floor-beams,and also between the floorbeam connecting angles and the floorbeam webs,which require the same number of rivets.
5. The rivet pitch at the ends of the floorbeam flanges is too great by .33 inches.
6. The intermediate girder stiffners fall far below Cooper's Specification.
7. The rivet spacing in the girder flanges of the first panel is .25 inches too large.

All but numbers 6&7 could be corrected by deep-ening the stringers and floorbeams, Nos.6 & 7 could be easily corrected without interfering with anything else.

If the bridge had been fabricated of medium steel instead of soft steel there is little doubt but that all the specifications would have been met with, except possibly in the case of the stiffeners. Stiffeners seldom come up to Cooper's Specifications in common practice.

The End.

Table of Contents.

****